Behaviour of Geogrid Reinforced Soil under Earthquake Loading

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Abstract

Energy dissipation in geological materials reinforced with geosynthetic is studied in this research. The behavior of building shallow foundation subjected to long period and short period earthquakes has been studied by numerical models prepared using MIDAS-GTS software. The models with multilayer geogrids below foundation are prepared. Time history analyses are then performed. Results are analyzed with the help of time history graphs and the three dimensional graphic display. Analyses are then done on the model of soil-geosynthetic-structure interaction with soil dynamic properties (stiffness) for different damping. The results obtained are interpreted and discussed.

Keywords

Geogrid; Pseudo Acceleration; Damping; Time History Analysis

Introduction

The earthquake causes destruction on earth surface that depends upon the amount of intensity of earthquake. Many researchers have worked on earthquake wave energy flow and distribution. The effect of soil-structure interaction and non-linear site response of soil, as well as the motion parameters such as acceleration, velocity and displacement were the focus of the several studies since different earthquakes have different intensities.

The effective acceleration which is less than the peak free-field ground acceleration, is most closely related to structural response and to damage potential of an earthquake. It is a function of the size of the loaded area, frequency content of the excitation, which in turn depends on the source of earthquake, and on the weight, embedment, damping characteristic, and stiffness of the structure and its foundation.

Earthquake loading is generally applied so rapidly that all but the most permeable of soils are loaded under undrained conditions. Soil properties that influence wave propagation and other low-strain phenomena include stiffness, damping, Poisson's ratio and density.

Most of the structures can be idealized as assemblage of discrete masses with discrete sources of stiffness. The geologic materials treated as continua and their responses to dynamic disturbances must be described in the context of wave propagation.

The effect of earthquake has studied by model field test, centrifuge test or by proto type laboratory model tests. The actual earthquake results are compared with the above tests and from that results the amount of structural damages predicted; and accordingly depend upon seismic demand the structural design can be done.

Energy Dissipation in Geological Materials

Seismic wave amplitudes are reduced as waves propagate through an elastic medium. Such reduction is consequence of energy losses in the soil, and that is named "attenuation". The attenuation characteristics can reveal unique information about lithology, physical state, and degree of rock saturation (Parrales, 2004). Seismic wave attenuation in geotechnical materials is a complex phenomenon resulting from the interaction of several mechanisms that contribute to the energy dissipation during dynamic excitation. Several definitions have been proposed as measures of energy dissipation in geological materials. Most are dimensionless parameters. The spreading of energy over an expanding area as the wave front propagates away from the source causes the amplitude of waves to attenuate with increasing distance from the source (geometric or radiation damping). Reflection and transmission of seismic waves at interfaces, mode

conversions, and scattering in non-homogeneous media also cause amplitudes to diminish which are collectively called apparent attenuation (Parrales, 2004).

Damping Ratio

In soil dynamics and geotechnical earthquake engineering, the parameter traditionally used as a measure of energy dissipation during harmonic excitation is the material damping ratio D. The damping ratio is defined as

$$D = \frac{\Delta E}{4\pi E}$$

where ΔE is the energy dissipated during one cycle at circular frequency ω ; and E is the maximum strain energy stored during that cycle.

Dissipation Factor

Geophysicist and seismologist often use the quality factor Q or its inverse, the dissipation factor Q-1 to describe material attenuation. The dissipation factor also called "specific attenuation factor" (Parrales, 2004) or specific dissipation function (Parrales, 2004) is defined as

$$Q^{-1} = \frac{\Delta E}{2\pi E} = 2D$$

An infinite Q means that there is no attenuation (Parrales, 2004).

Logarithmic Decrement

Another measure of attenuation is obtained from the logarithmic decrement d of a harmonic wave. Except for nonlinear behaviour near earthquake foci, seismic strains are small and seismic oscillations take place in the linear domain of elasticity. Attenuation of harmonic signals is therefore exponential, and the magnitude of the attenuation is describable by the exponential or logarithmic rates of decay. Experiments show that nonlinearity is introduced for strains in excess of 10^{-5} or 10^{-6}. Seismic strains are generally below this value. The linear region of excitation corresponds to attenuation factors which are independent of the amplitude of the excitation (Parrales, 2004).

The decay of the amplitude of vibrations is similar to that described for free vibration of a viscously damped system. The internal damping in soils is not considered to be the result of a viscous behaviour; nevertheless, the theory for a single-degree-of-freedom system with viscous damping is a useful framework to describe the effect of the damping which actually occurs in soils (Parrales, 2004).

The decay of free vibration of a single-degree-of-freedom system with viscous damping is described by the logarithmic decrement, which is defined as the natural logarithm of two successive amplitudes of motion (z1 and z2), or

$$\delta = ln\frac{z_1}{z_2} = \frac{2\pi D}{\sqrt{1-D^2}}$$

Coefficient of Attenuation

Yet another way to measure the attenuation is in terms of the coefficient of attenuation a. The coefficient of attenuation is related to the logarithmic decrement by

$$\delta = \frac{2\pi v\alpha}{\omega} = L\alpha$$

in which v is the velocity and L is the wave length of the propagating wave (Parrales, 2004).

The dissipation factor is related to the coefficient of attenuation by

$$Q^{-1} = \frac{2v\alpha}{\omega}$$

Soil Reinforcement

The results of experimental studies reported in the literature (Radhey Sharma et al. 2009) showed that the bearing capacity of soil improved when reinforced by geosynthetic and that better improvements were obtained when the reinforcement is placed within a certain depth (or influence depth) beyond which no significant improvement will occur. Different studies resulted in somewhat different specifications for reinforcement layouts. Combining the results from literature showed that: (i) the first reinforcement layer should be located close to the bottom of the footing at an optimum depth, *u*, of 0.2*B*–0.5*B* (*B* is the width of footing), (ii) the optimum vertical spacing, *h*, of reinforcement was found to be 0.2*B*–0.5*B*, (iii) The maximum total depth, *d*, of the reinforcement varied from 1.0*B* to 2.0*B*, (iv) the effective length, *l*, of the reinforcement was found to vary from 2.0*B* to 8.0*B*, (v) the geogrids with higher tensile modulus outperformed geogrids with lower tensile modulus, and (vi) geogrid reinforced soil foundation outperformed geotextile-reinforced soil foundation.

Different boundary conditions affect the behavior of reinforced soil:

Rigid boundary effect: when the depth of the first layer of reinforcement (u) is greater than a certain value, the reinforcement would act as a rigid boundary, and the failure would occur above the top layer reinforcement.

***Membrane effect*:** with the applied load, the footing and soil beneath the footing move downward; and the reinforcement is deformed and tensioned. Due to its stiffness, the curved reinforcement develops an upward force to support the applied load. A certain amount of settlement is needed to mobilize tensioned membrane effect, and the reinforcement should have enough length and stiffness to prevent it from failing by pulling out and tension.

***Confinement effect or lateral restraint effect*:** due to relative displacement between soil and reinforcement, the friction force is induced at the soil–reinforcement interface. Furthermore, the interlocking can be developed by the interaction of soil and geogrid. Consequently, lateral deformation or potential tensile strain of the reinforced soil is restrained. As a result, vertical deformation of soil is reduced. Since most soils are stress-dependent materials, improved lateral confinement can increase the modulus/compressive strength of soil, and thus improving the bearing capacity of reinforced soil.

Numerical Modelling

In this study the **MIDAS-GTS** software is used for numerical modeling. The thickness of the soil bed is 30 m and the total model dimensions are 58.4 m x 24 m. The rectangular foundation size selected is 1.8m x 3.2m for model from Rayhani & Naggar 2007 in which the height of the building is taken as h and r is the radius of circular foundation considering $\frac{h}{r} = 2.23$. The **TECH-FEB** biaxial geogrid is used as reinforcement. Different models such as foundation on unreinforced soil and reinforced soil having one, two, three and four geogrids are prepared. The first geogrid is provided at foundation depth and then each layer is at 0.5 m depth from the previous layer. The five different magnitudes and different time intervals earthquakes are applied at the soil base. The acceleration, velocity, displacement, stresses and forces are studied for soft soil and medium stiff soil.

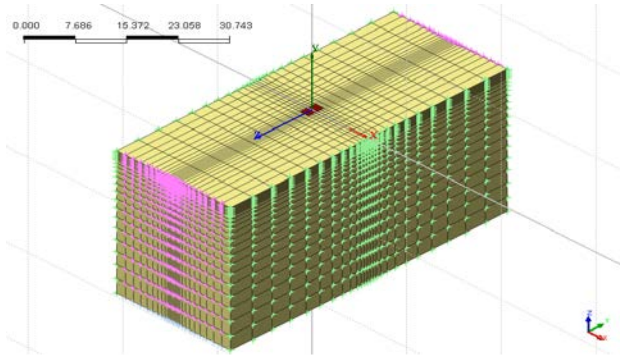

FIG.1 NUMERICAL MODEL FOR MEDIUM STIFF SOIL WITH FOUNDATION

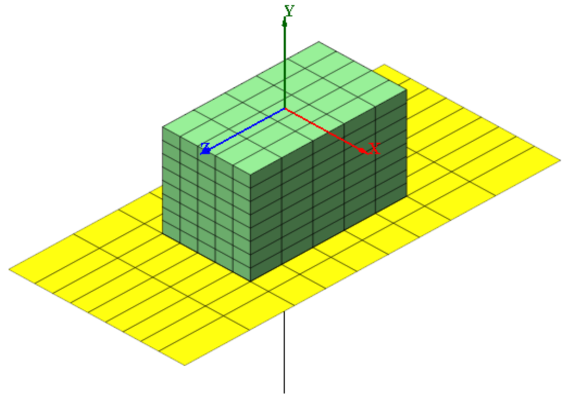

FIG.2 FOUNDATION ON GEOGRID

Methodology

For the present study, a commercial MIDAS-GTS (2012) finite element tool is used to analyze the numerical model of soil-geosynthetic-structure interaction. The numerical modelling is performed based on the collected input data. The analysis on soil input data for the model is collected from the early experimental centrifuge test data (Rayhani & Naggar 2008) and geosynthetic material as biaxial geogrid of polypropylene (PP) (*Tech Feb India Ltd*) is used. Initially, a three dimensional geometry is created using software tools. The attributes and properties of the system are then assigned. Appropriate constitutive relations are used to model the soil-geosynthetic-structure interaction system. Element map mesh generation is followed by the input of boundary conditions and loads. Eigen value analyses are to be done in order to obtain the natural frequency of vibration. Ground accelerations of varying magnitudes are applied to the system. Time history analyses are then performed. Results are analyzed with the help of time history graphs and the three dimensional graphic display. Analyses are then done on the model of soil-geosynthetic-structure interaction

with soil dynamic properties (stiffness). The results obtained are interpreted and discussed.

TABLE 1 SEQUENCE OF SHAKING EVENTS

Event	Yorba Linda	Kobe	Bhuj	Utterkashi	Sendai
Peak Base Acceleration (g)	0.016	0.04	0.5	2.28	150
Time (s)	7.24	15.5	16	4.46	81
Magnitude (Richter scale)	4	7	7	7	9

TABLE 2 GROUND AND STRUCTURAL MATERIAL PROPERTIES USED IN MODELLING

Name	Medium stiff Soil	Concrete Foundation	Bi-axial Geogrid Reinforcement
Model	Mohr Coulomb	Elastic	Geogrid-2D
Modulus of Elasticity(kN/m2)	45000	100000	5000000
Poisson's Ratio	0.42	0.25	0.4
Unit Weight(kN/m3)	15.95	25	0.5
Cohesion	90	0	0
Angle of Internal Friction	24	0	0
Thickness (m)	30	2	0.005
Size	58.4 m x 24 m	2B x 4B	4B x 6B

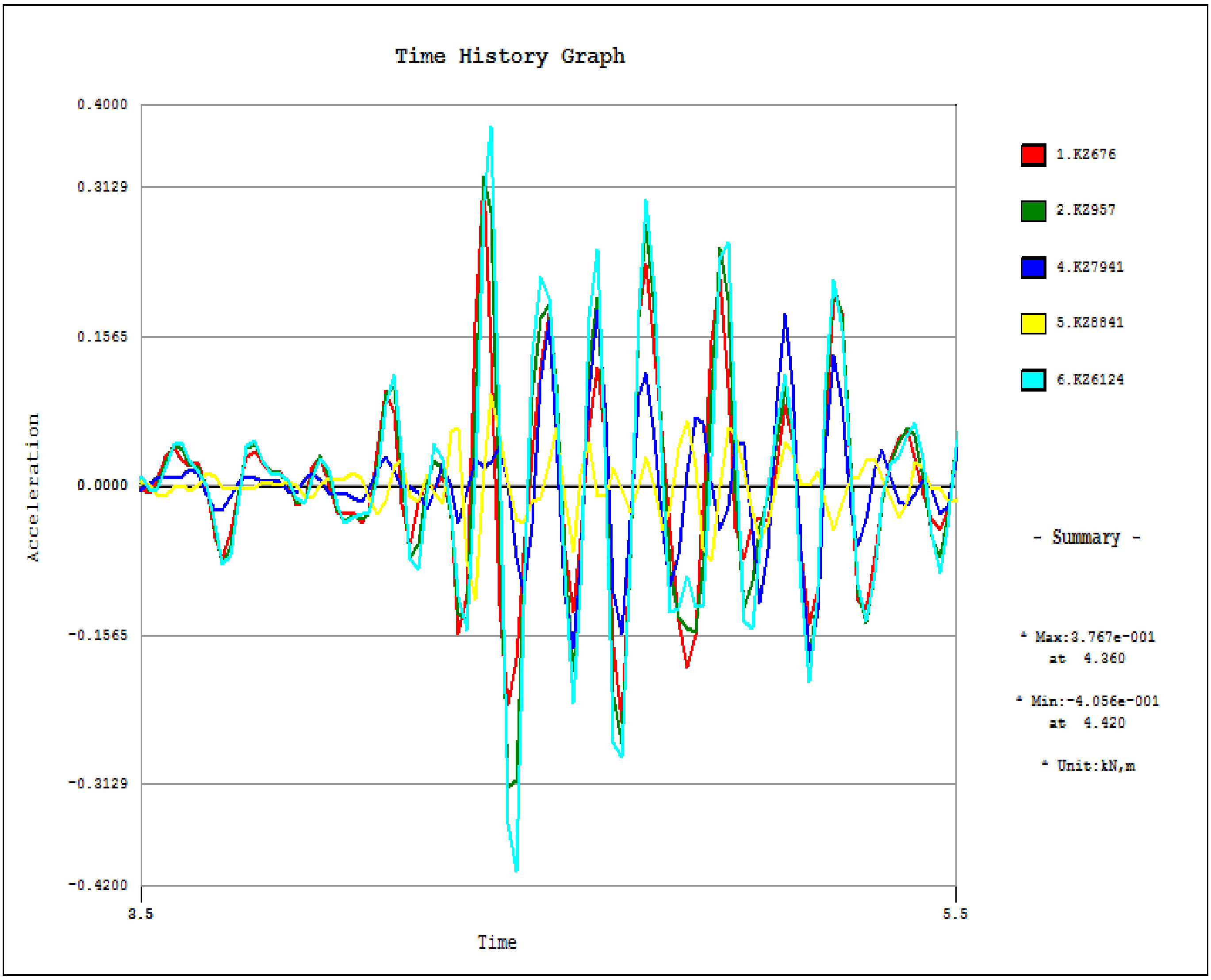

FIG.3 EFFECT OF WAVES AMPLITUDE FROM BASE OF SOIL TO THE TOP OF FOUNDATION FOR KOBE EARTHQUAKE

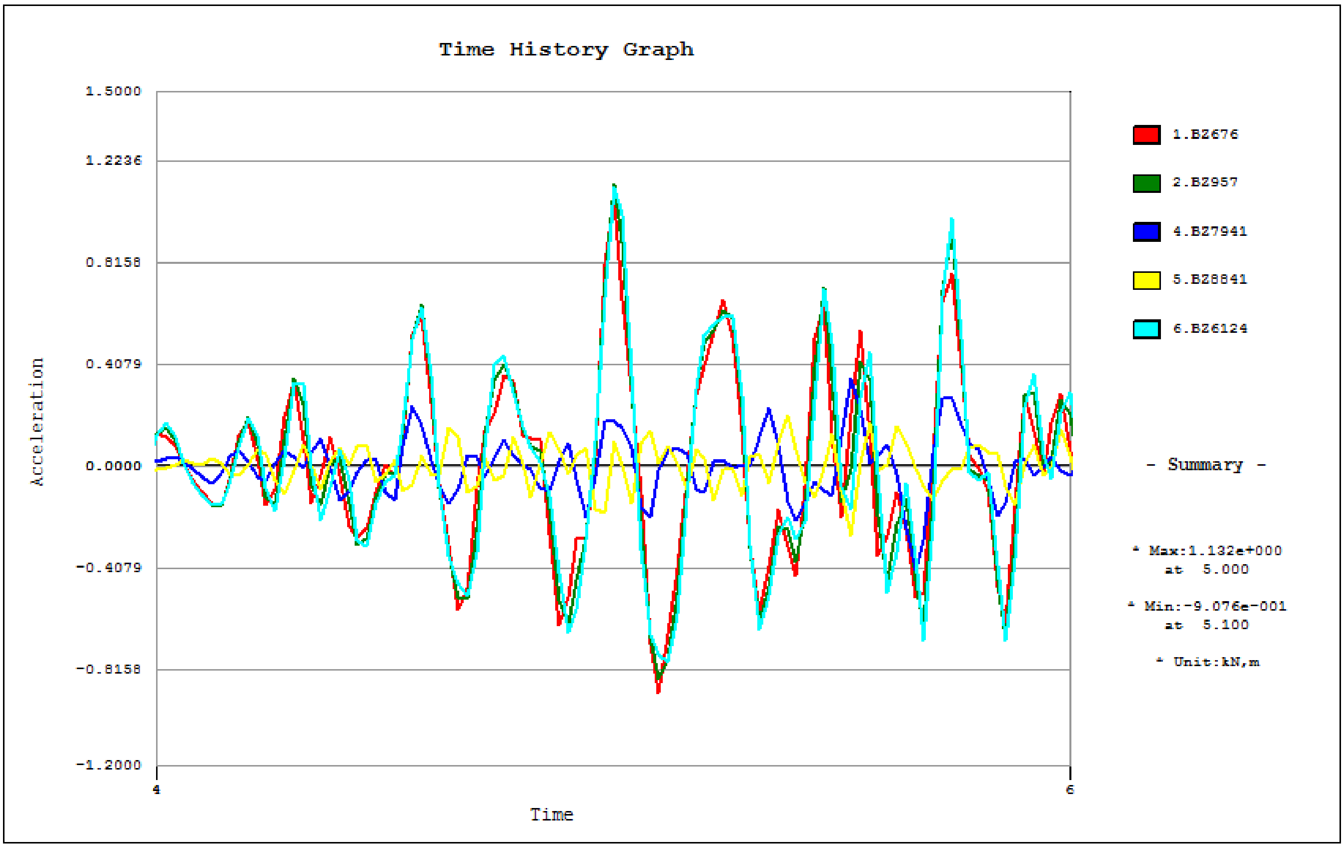

FIG.4 EFFECT OF WAVES AMPLITUDE FROM BASE OF SOIL TO THE TOP OF FOUNDATION FOR BHUJ EARTHQUAKE

FIG.5 EFFECT OF WAVES AMPLITUDE FROM BASE OF SOIL TO THE TOP OF FOUNDATION FOR SENDAI (TSUNAMI JAPAN)EARTHQUAKE

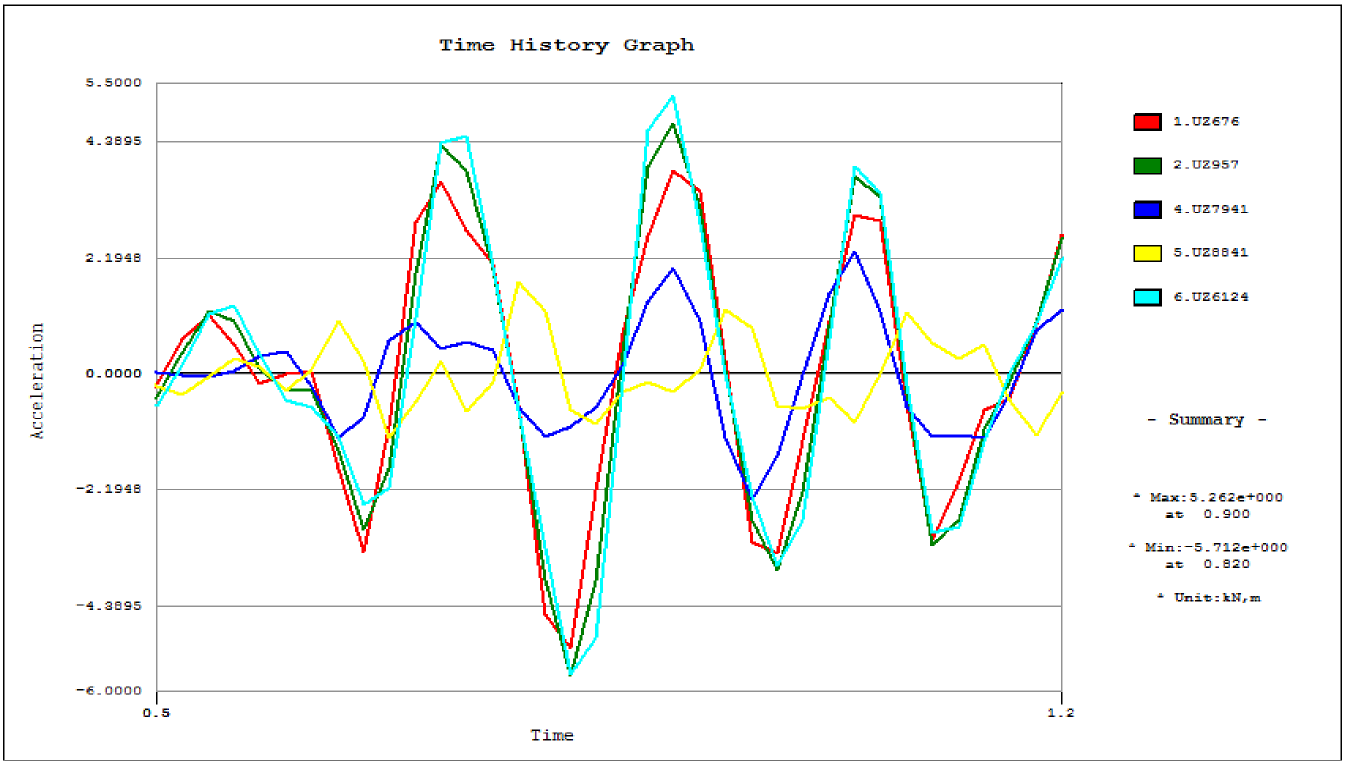

FIG.6 EFFECT OF WAVES AMPLITUDE FROM BASE OF SOIL TO THE TOP OF FOUNDATION FOR UTTERKASHI EARTHQUAKE

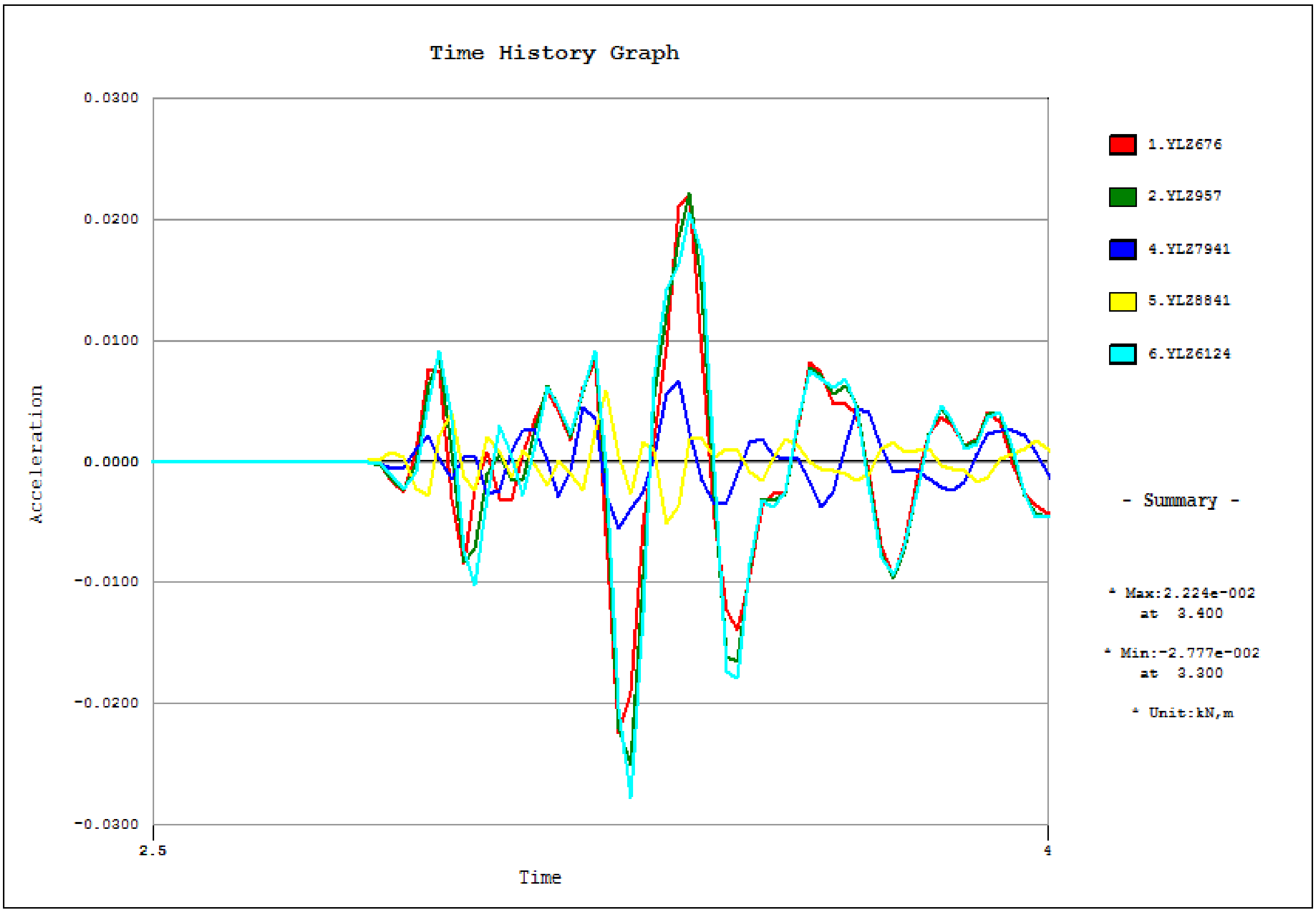

FIG.7 EFFECT OF WAVES AMPLITUDE FROM BASE OF SOIL TO THE TOP OF FOUNDATION FOR YORBA LINDA EARTHQUAKE

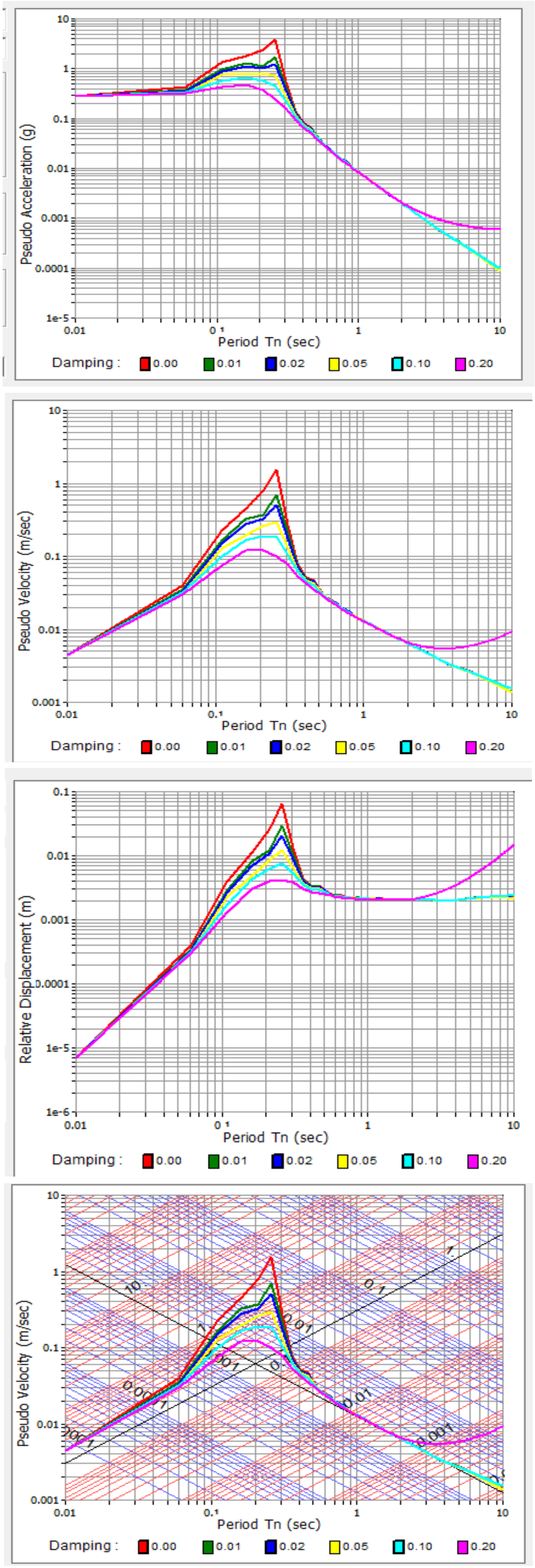

FIG. 8 RESPONSE OF MS0G UNDER KOBE EARTHQUAKE

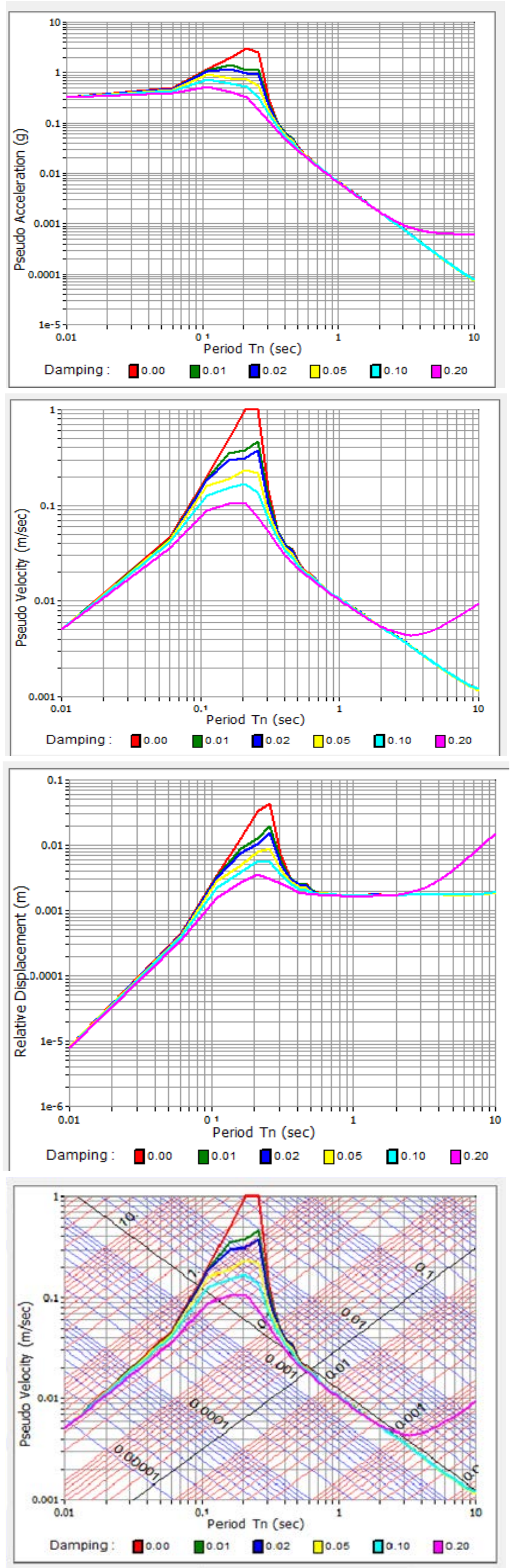

FIG. 9 RESPONSE OF MS1G UNDER KOBE EARTHQUAKE

Results and Discussion

The behavior and response of shallow foundation in medium stiff soil with and without geogrid under earthquake loading is studied by time history graphs. Fig 3 to fig 7 shows that maximum acceleration is experienced at geogrid level and then the acceleration reduced from bottom of foundation to the top of foundation. For long period earthquakes, the percentage reduction in acceleration is more than that in the short period earthquakes when the soil is reinforced below foundation. Figure 8 medium stiff soil without reinforcement (MS0G) and figure 9 medium stiff soils with one reinforcement (MS1G) show the response spectrum for Kobe earthquake. The similar results are also obtained for other shaking events.

In the presence of geogrid, there is logarithm decrease in pseudo acceleration, pseudo velocity and relative displacement but at the same time it is clear from graphs that there is increase in length of time period as peak point is widen in figure 9 compared to figure 8. Figure 9 indicates that the part of earthquake energy is resisted by geogrid present in soil mass.

Conclusions

The pseudo acceleration, pseudo velocity and relative displacement decrease due to reinforced soil structure interaction for different damping ratio which is clear from figure 8 and 9. Figure 3 to 7 show the behaviour of reinforced soil with foundation under long period and short period earthquake. Though the magnitude of Kobe, Bhuj and Utterkashi is same acceleration and time period affect the wave propagation in medium stiff reinforced soilto different degree.

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